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11.1 GENERAL

Deep foundation support systems have been in existence for a long time. The first known pile foundations consisted of rows of timber stakes driven into the ground. Timber piles have been found in good condition after several centuries in a submerged environment. Several types of concrete piles were devised at the turn of the twentieth century. The earliest concrete piles were cast-in-place followed by reinforced, precast, and prestressed concrete piling. The requirement for longer piles with higher bearing capacity lead to the use of concrete filled steel pipe piles in about 1925. More recently, steel H-piles were specified due to ease of fabrication, higher bearing capacity, and greater durability during driving.

The primary functions of a pile foundation are:

1. To transmit the load of the structure through a stratum of poor bearing capacity to one of adequate bearing capacity.
2. To eliminate objectionable settlement.
3. To transfer loads from a structure through easily erodible soils in a scour zone to a stable underlying stratum.
4. To anchor structures subjected to hydrostatic uplift or overturning forces.
5. To resist lateral loads from earth pressures, as well as external forces.
6. To serve as a retaining structure when installed in groups or in a series of overlapping piles.
7. To improve the load bearing capacity of a soil.

(1) Bearing Capacity of Pile Foundations

A pile foundation transfers load into the underlying strata by either skin friction, point bearing, or a combination of skin friction plus point bearing. Any driven pile will develop some amount of both skin friction and end bearing. However, a pile that receives the majority of its support capacity by friction or adhesion from the soil along its shaft is referred to as a friction pile, whereas a pile that receives the majority of its support from the resistance of the soil near its tip is an end bearing pile.

The design pile capacity is the maximum load the pile can support without exceeding the allowable movement. When considering design capacity, one of two items may govern the design; the ultimate Geotechnical capacity or the structural capacity of the pile section. This chapter focuses on computing the Geotechnical capacity of a pile.

The load applied to a single pile is carried jointly by the soil beneath the tip of the pile and by the soil around the shaft. The maximum load that the pile can support (the pile capacity) is:

$$Q = Q_p + Q_s \text{ -where:}$$

Q = pile capacity

Q_p = point resistance

Q_s = shaft resistance

This is also shown in the following Figure 11.1.

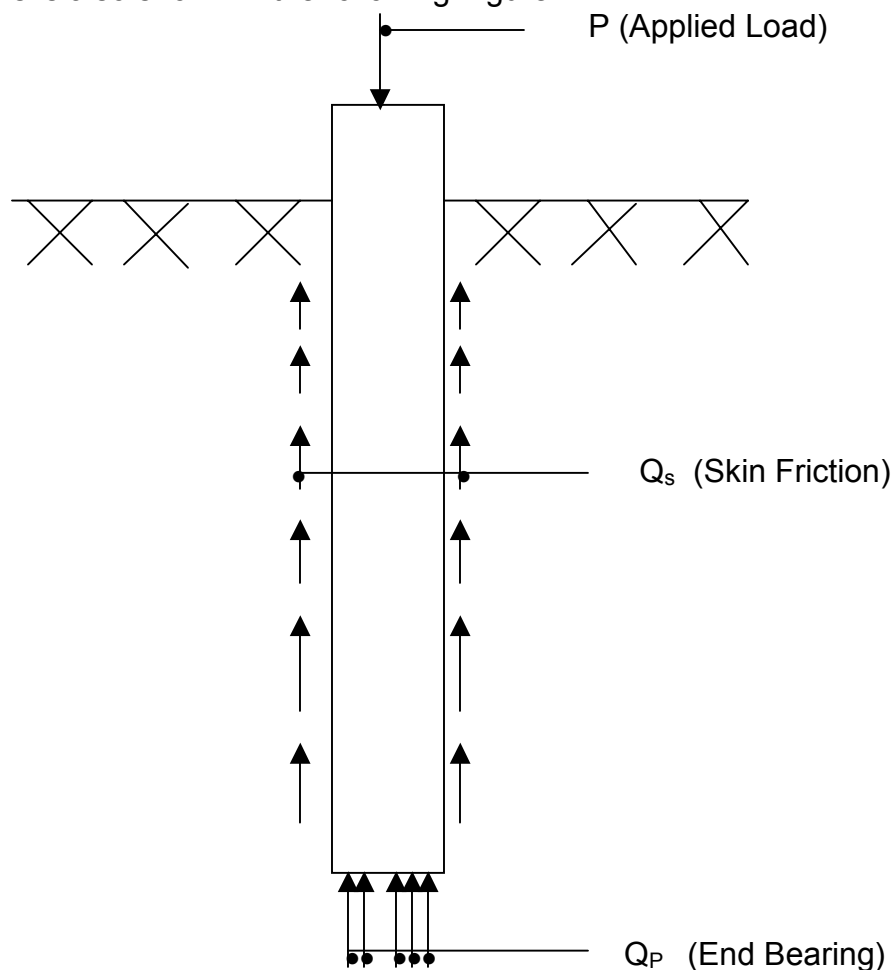


FIGURE 11.1

A. Shaft Resistance

The shaft resistance of a pile is estimated by summing the frictional resistance developed in each of the different soil strata. These formulas take the form of:

Total shaft resistance for **non-cohesive** soils can be calculated from the equation: (Nordlund's Method)

$$Q_s = \sum K_\delta C_F P_d \sin \delta C_d \Delta d \text{ where:}$$

Q_s = Total skin friction capacity

K_δ = Dimensionless factor relating normal stress and effective overburden pressure

C_F = Correction factor for K_δ when $\delta \neq \phi$

P_d = Effective overburden pressure at the center of depth increment 'd'

δ = Friction angle on the surface of sliding

C_d = Pile perimeter

d = Depth increment below ground surface

Total shaft resistance for **cohesive** soils can be calculated from the equation:

$$Q_s = \sum C_a C_d D \text{ where:}$$

Q_s = Total skin friction capacity

C_a = Pile adhesion factor

C_d = Perimeter of pile

D = Pile segment length

Average values of friction for various soils are given in Tables 11.1 and 11.2. The values given are average ranges and are intended to give orders of magnitude only. Other conditions such as layering sequences, drilling information, ground water, thixotropy, clay sensitivity, etc., must be evaluated by experienced Geotechnical engineers and analyzed using principles of soil mechanics.

Skin friction values are dependent upon soil texture, overburden pressure and soil cohesion, but tend to increase with depth. However, experience in Wisconsin has shown that skin friction values in non-cohesive materials reach constant final values at depths of 15 to 25 pile diameters in loose sands and 25 to 35 pile diameters in firm sands.

COHESIVE MATERIAL

<u>Soil Type</u>	<u>qu ⁽¹⁾</u>		<u>Skin Friction (S.F. = 2)</u>	
	<u>tons/sf</u>	<u>kPa</u>	<u>(psf)</u>	<u>kPa</u>
Very soft clay	0-.25	0 - 24		
Soft clay	.25-.50	24 - 48	100-300	5 - 15
Medium clay	.50-1.0	48 - 96	250-450	12 - 21
Stiff clay	1.0-2.0	96 - 192	550-950	26 - 45
Very stiff clay	2.0-4.0	192 - 384	1000-1500	48 - 72
Hard clay	4.0	384	1500-2000	72 - 96
Silt			50-250	2 - 12
Silty clay			200-400	10 - 19
Sandy clay			200-400	10 - 19
Sandy silt			300-500	14 - 24
Dense silty clay			450-750	21 - 36

TABLE 11.1**GRANULAR MATERIAL**

<u>Soil Type</u>	<u>N ⁽²⁾</u>	<u>Skin Friction (S.F. = 2)</u>	
		<u>(psf)</u>	<u>(kPa)</u>
Very loose sand and silt or clay	0 - 6	25-75	1 - 4
Medium sand and silt or clay	6 - 30	200-300	10 - 14
Dense sand and silt or clay	30 - 50	300-400	14 - 19
Very dense sand and silt or clay	over 50	400-500	19 - 24
Very loose sand	0 - 4	350-850	17 - 41
Loose sand	4 - 10	350-850	17 - 41
Firm sand	10 - 30	350-850	17 - 41
Dense sand	30 - 50	350-850	17 - 41
Very dense sand	over 50	350-850	17 - 41
Sand and gravel		500-1500	24 - 72
Gravel		750-1750	36 - 84

TABLE 11.2

- (1) Unconfined Compression Strength
 (2) Standard Penetration Value (AASHTO T206)

In computing shaft resistance, the method of installation must be considered as well as the soil type. The method of installation significantly effects the degree of soil disturbance, the lateral stress acting on the pile, the friction angle and the area of contact. Shafts of prebored piles do not always fully contact the soil, therefore the effective contact area is less than the shaft area. Driving a pile in granular material densifies the soil and increases the friction angle. Driving also displaces the soil laterally and increases the horizontal stress acting on the pile. Disturbance of clay soils from driving can break down soil structure and increase pore pressures which greatly decrease soil strength. However, some or all of the strength recovers following reconsolidation of the soil due to a decrease in excess pore pressure over time. Use the initial soil strength values for design purposes. The type and shape of a pile also affects the amount of skin friction developed. Refer to Section 11.2.

Negative skin friction, in which soil pulls down the pile instead of supporting the load, can occur when settlement of the soil through which the piling is driven takes place. It has been found that only a small amount of settlement is necessary to mobilize these additional pile (drag) loads. This settlement occurs due to consolidation of softer soil strata caused by such items as increased embankment loads (i.e. earth fill) or a lowering of the existing groundwater elevation. When this condition is present, the allowable pile loads are reduced to compensate for the increase in soil overburden load, or consolidation is allowed to occur before driving piling. Other alternatives are to preauger the piling, too drive the pile to bearing within a permanent pipe sleeve that is placed from the base of the substructure unit to the bottom of the soft soil layer(s), to coat the piles with bitumen above base of the compressible soil strata or to use proprietary materials to encase the piles (within fills).

Uplift forces may also be present, both permanently and intermittently, on a pile system. Such forces may occur from hydrostatic uplift or cofferdam seals, ice uplift resulting from ice grip on piles and rising water, wind uplift due to pressures against high structures, frost uplift, etc. In the absence of pulling test data, friction values (with appropriate reductions) on the order of magnitude as given in Tables 11.1 and 11.2 can be assigned. Values from this table are average ranges only and are to be used judiciously.

Generally, the type of pile with the largest perimeter is the most efficient in resisting uplift forces. Factors of safety to be used depend upon the expected frequency and duration of the uplift forces. AASHTO Specifications limit the uplift resistance to a maximum of 33% of the ultimate downward friction capacity for uplift conditions. Group action is also considered, refer to Section 11.1(1)D.

B. Point Resistance

The point resistance (end bearing) capacity of a pile is estimated from modifications to the bearing capacity formulas developed for shallow footings.

These formulas take the form of:

Point resistance for **non-cohesive** soils can be calculated from the equation: (Thurman's Method)

$$Q_p = A_p \alpha P_d N'_q \text{ where:}$$

Q_p = End bearing capacity

A_p = Pile end area

α = Dimensionless factor dependent on depth-width relationship

P_d = Effective overburden pressure at the pile point (≤ 3000 psf)

N'_q = Bearing capacity factor

$$Q_p \text{ must also be } \leq Q_{lim} = A_p q_{lim}$$

Q_{lim} = Limited capacity of pile in granular material

q_{lim} = Limiting unit point resistance based on angle of internal friction

Point resistance for **cohesive** soils can be calculated from the equation:

$$Q_p = 9C_u A_p \text{ where:}$$

Q_p = End bearing capacity

C_u = Adhesion of the soil to the pile

A_p = Pile end area

This equation represents the maximum value of point resistance for cohesive soils. This value is often considered to be zero because substantial movement of the pile tip (1/10 of the pile diameter) is needed to mobilize end bearing capacity. This amount of tip movement seldom occurs after installation.

An end bearing pile surrounded by soil is not a structural member like a column. Both experience and theory demonstrates that there is no danger of an end bearing pile buckling due to inadequate lateral support if it is surrounded by even the very softest soils. Therefore, pile stresses can exceed allowable column stresses.

C. Lateral Load Capacity

Recommended values of lateral resistance for battered or vertical piles used by the Structures Design Office in lieu of a detailed analysis are:

Soil Conditions	Timber		10-3/4" (273 mm) CIP		12" Precast/ 12-3/4" (320 mm) CIP		Steel-H	
Poor	4k	18 kN	7k	31 kN	8k	36 kN	7k	31 kN
Average	5	22 kN	9	40 kN	11	49 kN	10	44 kN
Good	5	22 kN	9	40 kN	11	49 kN	15	67 kN

Structures supported by single piles or pile groups are frequently subjected to lateral forces from lateral earth pressure, live load forces, wave action, ice loads and wind forces. Piles subjected to lateral forces must be designed to meet allowable stress and deflection criteria to prevent impairment or premature failure of the foundation or superstructure. To solve the soil-structure interaction problems, the designer determines the following:

1. Characteristics of the pile including:
 - a. Stiffness of the pile.
 - b. Rotation restrictions imposed on the pile top by the cap.
 - c. Maximum bending moment imposed on the pile and the distribution of the bending moment along the pile length.
 - d. Probable points of fixity on the pile.
2. Stiffness of the soil.
3. Allowable deflection of the pile permitted by the superstructure.

Many theories for lateral load capacity are based upon Terzaghi's 1955 theory of subgrade reaction. For a more detailed analysis of lateral loads and displacements refer to the listed FHWA design references or a Geotechnical Engineering book. There are usually at least two analysis methods; one for short, stiff piles and one for long, flexible piles. In addition, there are computational differences for piles with rigid caps and piles with flexible caps.

D. Pile Group Action

Friction piles in group action do not carry as much load per pile as an individual pile acting alone due to overlapping zones of pressure around each pile. The amount of reduction per pile depends upon the size and shape of the pile group, as well as the size, spacing, and length of the piles. No reduction due to grouping occurs with end bearing piles. For combined end bearing and friction piles, only the load carrying capacity of the frictional portion is reduced. Experience in Wisconsin indicates that in most thixotropic clays where piles are

driven to a hammer bearing as determined by dynamic formulas, reduction for pile group action is not required. Since reductions are not usually made, bridge computer programs do not include an efficiency formula for pile group reduction.

In special cases, at sites where piles are driven in groups into plastic materials the designer should consider a reduction for pile group action. The Converse-Labarre efficiency formula is suggested in the AASHTO Specifications. This method is derived under the assumption that the area of a pile available for developing shear is reduced by the influence of adjacent piles in the same row and by the closest pile of the adjacent row. However, it disregards pile length, battered piles capped by a seal, and distances of embedment in friction strata. When applying this formula, consider battered piles by increasing the value "S" to "S" plus the amount of batter at the third point from the top of pile. The reduced pile capacity is compared to Group I axial loading for adequacy.

E. Conditions Involving Short Pile Lengths

WisDOT policy is to use piles driven a distance of 10 feet (3 meters) or greater below the original ground surface. Pile penetrations of less than 10 feet (3 meters) are allowed if prebored at least 3 feet into solid rock or 5 feet into sandstone. Concern exists that short pile penetration in foundation materials of variable consistency may not adequately restrain lateral movements of substructure units. If conditions detailed in the Site Investigation Report clearly indicate that minimum pile penetration cannot be achieved, preboring should be included as a pay quantity. If there is a potential that the need for preboring is questionable, do not include it in the plan documents. Piles which are not prebored into rock must not only meet the 10 ft. min. pile penetration criteria, but must also have at least 5 ft. of penetration through material with a blow count of at least 7 blows/foot. After placement of piles into prebored holes, the piling should be 'driven' to ensure they are not founded on incompetent materials. The annular space between the cored holes and piling should then be filled with concrete.

Foundations without piles (spread footings) should be given primary consideration at sites where pile penetrations of less than 10 feet (3 meters) are anticipated. The economics of the following choices should be investigated:

1. Design for a spread footing founded at a depth where the foundation material is adequate. Key the footing 6 inches (150 mm) into sound rock for lateral stability.
2. Excavate to an elevation where foundation material is adequate and backfill to the bottom of footing elevation with suitable granular material or concrete.

If a substructure unit is located in a stream, consideration should be given to the

effects of the anticipated stream bed scour when selecting the footing type. Pile length computations should not incorporate pile resistance developed within the scour zone. The pile cross-section should also be checked to ensure it can withstand the driving necessary to penetrate through the anticipated scour depth and reach design capacity plus the frictional capacity within the scour zone.

If boulders or cobbles are anticipated within the estimated length of the pile, consideration should be given to increasing the CIP pile shell thickness to reduce the potential of pile damage due to high driving stresses. Another alternative is to investigate the use of HP piles at the site.

F. Pile Spacing

Arbitrary pile spacing rules specifying maximums and minimums are extensively used in foundation design. Proper spacing is dependent upon length, size, shape, and surface texture of piles as well as soil characteristics. A wide spacing of piles reduces heaving and possible uplifting of the pile, damage by tension due to heaving, and the possibility of crushing from soil compression. Wider spacings more readily permit the tips of later driven piles in the group to reach the same depths as the first piles and result in more even bearing and settlement. Large horizontal pressures are created when driving in a relatively uncompressible strata and damage may occur to piles already driven if piles are too closely spaced. In order to account for this, a minimum center-to-center spacing of 2- 1/2 times the pile diameter is often required. AASHTO Specifications call for a center-to-center pile spacing of not less than 2'-6" (750 mm) or 2.5 pile diameters (widths). WisDOT maximum pile spacing is 8'-0", based on abutment/pier structural designs. See the Pier Chapter for criteria on battered piles in cofferdams. The distance from the side of any pile to the nearest edge of footing shall be greater than 9". Piles shall project at least 12" into the footings.

(2) Pile Driving Hammers

Pile driving hammers are generally powered by compressed air, steam pressure, or diesel units. The diesel hammer, a self-contained unit, is the most popular due to its compactness and adoption in most construction codes. Also, the need for auxiliary power is eliminated and the operation cost is nominal. Vibratory and sonic type hammers are employed in special cases where speed of installation and/or noise from impact is prohibited. The vibrating hammers convert instantly from a pile driver to a pile extractor by merely tensioning the lift line.

The hammer is raised and allowed to fall either by gravity or with the assistance of power. If the fall is due to gravity alone, the hammer is referred to as single-acting. The single-acting hammer is suitable for all types of soils but is most effective in penetrating heavy clays. The major disadvantage is the slow rate of driving due to the relatively slow rate of blows from 50 to 70 per minute.

Wisconsin construction specifications call for a minimum hammer weight depending on the required final bearing value of the pile being driven. In order to avoid damage to the pile, the fall of the gravity hammer is limited to 10 feet (3 meters) with an absolute maximum of 15 feet (4.5 meters).

If power is added to the downward falling hammer, the hammer is referred to as double-acting. This type of hammer works best in sandy soil but also performs well in clay. Double-acting hammers deliver 100 to 250 blows per minute, which increases the rate of driving considerably over the single-acting hammers. Wisconsin construction specifications call for a rated minimum energy of 15 percent of the required bearing of the pile. A rapid succession of blows at a high velocity can be extremely inefficient, as the hammer bounces on heavy piles.

Differential-acting hammers overcome the deficiencies found with both single- and double-acting hammers by incorporating higher frequency of blows and more efficient transfer of energy. The steam cycle, which is different from that of any other hammer, makes the lifting area under the piston independent of the downward thrusting area above the piston. Sufficient force can be applied for lifting and accelerating these parts without affecting the dead weight needed to resist the reaction of the downward acceleration force. The maximum delivered energy per blow is the total weight of the hammer plus the weight of the downward steam force times the length of the stroke. Differential-acting hammers are becoming increasingly popular due to their economy, efficiency and reliability.

Generally the contractor's selection of the pile hammer is dependent on the following:

1. The hammer weight and rated energy are selected on the basis of supplying the maximum driving force without damaging the piles.
2. The hammer types dictated by the construction specification for the given pile type.
3. The hammer types available to the contractor.
4. Special situations such as sites adjacent to existing buildings that require consideration of vibrations generated from the driving impact or noise levels.
5. The subsurface conditions at the site.
6. The required final capacity of the piles.

Most specifications require the heads of all piling to be protected by caps during driving. The pile cap serves to protect the pile as well as modulate

the blows from the hammer which assist in eliminating high inefficient hammer forces. When penetration per blow is used as driving criteria, constant cap-block material characteristics are required. The cap-block characteristics are also assumed constant for all empirical formula computations to determine the rate of penetration equivalent to a particular dynamic resistance.

(3) Pile Driving Formulas

Formulas used to estimate the bearing capacity of piles are of four general types: empirical, static, dynamic and wave equation.

Empirical formulas are based upon tests under limited conditions and are not suggested for general use.

Static formulas are based on soil stresses and try to equate skin friction and end bearing to the load-bearing capacity of the piles.

Dynamic pile driving formulas are based upon the theory that ultimate carrying capacity is equal to the ultimate driving resistance. These formulas are derived starting with the relation:

$$\text{Energy input} = \text{energy used} + \text{energy lost}$$

The energy used equals the driving resistance times the pile movement. Thus by knowing the energy input and estimating energy losses, driving resistance can be calculated from observed pile movements. Numerous dynamic formulas have been proposed. They range from the simpler *Engineering News Formula*, to the more complex *Hiley Formula*. The State of Wisconsin, Standard Specifications call for use of a modified Engineering News Formula to determine pile capacity during installation.

Because of the difficulty of evaluating the many energy losses involved with pile driving, these dynamic formulas can only approximate pile driving resistance. These approximate results can be used as a safe means of determining pile length and bearing requirements. Despite the obvious limitations, the dynamic pile formulas take into account the best information available and do have considerable utility to the engineer in securing reasonably safe and uniform results over the entire project.

The wave equation is based upon the theory of longitudinal wave transmission. This theory, proposed by Saint-Venant a century ago, did not receive widespread use until the advent of computers, due to its complexity. The wave equation can predict impact stresses in a pile during driving and estimate static soil resistance at the time of driving by solving a series of simultaneous equations. The advantages of this method are adaptation to any shape pile, the frictional and viscous resistance of the soil, and the point resistance of the pile. The effect of the hammer and cushion block can be included in the computations.

WISDOT presently utilizes a Pile Driving Analyzer (PDA) in an advisory capacity for evaluating if sufficient pile penetration has been achieved to develop the design capacity, if pile damage may have occurred, or if a driving system is performing satisfactorily.

The PDA provides a method of dynamic pile testing both for pile design and construction control. Testing is accomplished during pile installation by attaching reusable strain transducers and accelerometers directly on the pile. Piles can be tested while being driven, or during restrike. The instrumentation mounted on the pile allows the measurement of force and acceleration signals for each hammer blow. This data is transmitted to a small field computer for processing and recording on a magnetic data tape for a permanent record. Data is also fed into an oscilloscope to observe the dynamic response of the pile to driving. Calculations made by the computer, based upon one-dimensional wave mechanics, provide an immediate printout of maximum stresses in the pile, energy transmitted to the pile, and a prediction of the ultimate pile bearing capacity for each hammer impact. Monitoring of the force and velocity wave traces with the oscilloscope during driving also enable detection of any structural pile damage that may have occurred. Printouts of selected force and velocity wave traces are also made to provide additional testing documentation. The Pile Analyzer can be used on all types of driven piles with any impact type of pile driving hammer, but is most often used for displacement piles.

(4) Pile Investigation

Test piles are employed at a project site for two purposes:

1. For test driving, to determine the length of timber pile required prior to placing purchasing orders.
2. For load testing, to verify actual pile capacity versus computed design capacity.

A. Driving of Test Piles

If timber piles are called for on Wisconsin Bridge plans, test piles are required to determine the length of piles to be ordered. Specify lengths sufficiently longer than the estimated test pile lengths (about 50 percent longer) to assure getting the required bearing capacity. Test piles are located so they can be used in the finished structure. If test piling cannot be incorporated in the substructure unit, the test piling is removed at least 2 feet (600 mm) below the stream bed or finished ground line.

Test piles are not required for spliceable types of piles. Previous experience indicates that contractors typically order total plan

quantities for cast-in-place or steel-H piling in 60 foot (18 meter) lengths. The contractor uses one of the driven structure piles as a test pile at each designated location.

Test piling should be driven near the location of a soil boring where the soil characteristics are known and representative of the most unfavorable conditions at the site. The test pile must be exactly the same type and dimension as the piles to be used in the construction and is installed by the same equipment and manner of driving. A penetration record is kept for every 1 foot (300 mm) of penetration for the entire length of pile. This record may be used as a guide for future pile driving on this project. Any pile encountering a smaller resistance is considered as having a smaller bearing capacity than the test pile.

Pile 'set up' has been found to occur in some fine-grained soils in the state. Set up occurs when the porewater pressures generated by pile driving operations dissipate over time, increasing the soil adhesion to the pile. Set up values should not be included in pile design unless pre-construction load tests are conducted to determine actual set up potential. Incorporating set up in the design of pile capacity requires a set-off during installation of the pile to allow this porewater dissipation to occur. The piling must then be remobilized (restruck) after a specified time to determine the amount of set up it experienced. It is more costly to drive piling when this waiting period is required on a project. Generally a certain percentage (10-20%) of the total number of piles is selected for restrike (also called retap). Typical special provisions have been developed for use on projects incorporating aspects of pile set up.

If a rest period is allowed before application of loads on test piles in clays or silts, it is usually a few days but in no case less than twenty four hours (48 - 72 hours recommended) in order to permit the disturbed soil to regain most of its strength. The Geotechnical Section recommends twenty-four hours for technical and economic reasons.

B. Load Testing of Piles

A pile load test is usually conducted to furnish information to the Geotechnical engineer to develop design criteria or obtain test data to substantiate design loads. A load test is the only reliable method of determining the bearing capacity of a single pile, but it is expensive and can be quite time consuming. The decision to embark on an advance test program is based upon the scope of the project and the

complexities of the foundation conditions. Such test programs often result in substantial savings in foundation costs, which can more than offset the test program cost.

Pile load testing usually involves the application of a direct axial load to a single vertical pile. However, load testing can involve uplift or axial tension tests, lateral tests applied horizontally, group tests or a combination of these applied to battered piles. Most static test loads are applied with hydraulic jacks reacting against either a stable loaded platform or a test frame anchored to reaction piles.

The basic information to be developed from the pile load test is usually the deflection of the pile head under the test load. Movement of the head is caused by elastic deformation of the piles and the soil. Soil deformation may cause undue settlement and must be guarded against. The amount of deformation is the significant value to be obtained from load tests, rather than the total downward movement of the pile head. Load tests are typically performed by loading to a given deflection value. Further load test information can be obtained from AASHTO test methods or various Geotechnical textbooks.

It is impractical to test every pile on a project, thus test results can be applied to other piles, providing that the following conditions exist:

1. The other piles are of the same type, material, and size as the test piles.
2. Subsoil conditions are comparable to those at the test pile locations.
3. Installation methods and equipment used are the same as, or comparable to, those used for the test piles.
4. Piles are driven to the same penetration depth or resistance or both as the test piles to compensate for variations in the vertical position and density of the bearing strata.

11.2 SELECTION OF PILE TYPES

The selection of a pile type for a given foundation is made on the basis of soil type, stability under vertical and horizontal loading, long term settlement, required method of pile installation, cost comparison, and length of pile. Frequently more than one type of pile meets the physical and technical requirements for a given site. The performance of the entire structure controls the selection of the foundation. Primary consideration in choosing a pile type is the evaluation of the foundation materials and the selection of the substratum that provides the best foundation support.

Where piers are located in water where the streambed is composed of loose to medium dense granular soil, and shifting of this material and scour is possible, piling is generally used, even though the material may provide adequate support without piling. In some cases, it is advisable to place footings at greater depths than minimum and specify a minimum pile penetration to guard against excessive scour beneath the footing and piling. Pile adhesion (skin friction) within the maximum depth of scour is assumed to be zero. When deep scour depths are estimated, this area of lost frictional support must be taken into account in the pile driving operations and capacities.

When competent bearing soils are not present near the ground surface, structure loads must be transferred to a deeper stratum by using piles or drilled shafts (caissons). Drilled shafts are generally large diameter, cast-in-place, open ended, cased concrete piles which are designed to carry extremely heavy loads. Drilled shafts can be the most economical at sites where foundation loads are carried to bearing on dense strata or bedrock. They are also cost effective in water crossings with very shallow bedrock, where cofferdams are difficult or expensive to construct.

Subsurface conditions at the structure site also affect pile selection and details. The presence of artesian water conditions, soft compressible soils, cobbles and/or boulders, loose/firm uniform sands, deep water, etc. all influence the selection of the optimum type of pile for deep foundation support. For instance, WisDOT has experienced 'running' of displacement piling in certain areas that are composed of uniform, loose sands. The Department has also experienced difficult driving of displacement piles in denser sands within cofferdams as consecutive piles are driven, due to compaction of the sands during pile installation within the cofferdam footprint.

Also, environmental factors can play an important role in pile selection. Examples are piles located in areas of potential high corrosion, those subject to bacterial corrosion, abrasion due to moving debris or ice, or wave action causing alternate wetting and drying and ultimate deterioration. Piles located in strong water currents can be subject to gradual erosion of the pile material due to scouring by abrasive river sediment. Alkaline soils or strong chemicals in rivers or streams can adversely affect concrete piles. Steel piles can suffer serious electrolysis deterioration if placed in an environment near stray electrical currents. Cast-in-place concrete piling should be used on all structure widenings where

displacement type piling are required. Timber piling are not to be used, even if they were used on the original structure.

Tapered steel displacement piling are proprietary and can be used for foundation materials consisting of loose to medium-dense granular soils for efficient transfer of loads along the surface of the piles by friction. The piles may need to be installed with the aid of water jetting if the granular soils are in a very compact state. Straight-sided pilings are recommended for foundation materials consisting of cohesive soil underlain by a granular stratum to develop the greatest skin friction along the piles and point bearing at the tip of the piles. Steel H-piles or open-ended pipe piles are best suited for driving through material obstructions or fairly competent layers, to bedrock. Foundations such as pier bents, which may be subject to large lateral forces when located in either deep or swiftly moving water or both, require piles that can sustain large bending forces. Precast, prestressed concrete piles are the best suited for high lateral loading conditions, but are seldom used on Wisconsin transportation projects.

Another consideration in pile and hammer selection is the presence of surrounding structures that may be damaged due to high vibration levels generated by pile driving operations. Pile driving operations can cause ground displacement, soil densification and other factors that can damage nearby buildings, structures, and/or utilities. Whenever pile-driving operations pose the potential for damage to these features (usually when they are located within approximately 100 feet), a vibration-monitoring program should be implemented. This program consists of reviewing pile driving plan submittals, conducting pre-driving and post-driving condition surveys and conducting the actual vibration monitoring with an approved seismograph. A special provision for implementing a vibration monitoring program is available and should be used on projects whenever pile driving operations pose a potential threat to nearby buildings, structures and/or utilities. Contact the Geotechnical Engineer for further discussion and assistance.

(1) Timber Piles

Current Design practice is to limit the use of timber piles. There are environmental concerns about the preservatives leaching into the groundwater. In addition, Cast-in-place steel piling are comparable in cost to timber piling, but have approximately twice the load carrying capacity. The steel piling lengths are easily adjustable in the field whereas problems have occurred with timber piling being too short at times or sometimes being too long. For timber piles which are too long, payment includes the excess length removed. Timber piling also requires a test pile which is not required on projects using steel piling.

Timber piles are available in lengths ranging from 30 to 60 feet (9 to 18 meters) with a maximum capacity of 30 tons (266 kN). Timber piles are classified as a displacement type with the majority of the pile load being carried by the skin friction developed between the pile and soil. It is standard practice to remove the bark

from the wood to eliminate a decomposed film which would ultimately create a plane of weakness. Timber piles are susceptible to damage in hard driving.

Timber piles are subject to deterioration caused from decay, insect attack and abrasion. Decay can be minimized if the wood is kept dry or entirely embedded in and cut off below ground water level. Reasonable protection against decay and insect attack can be achieved by impregnating the wood ('treating') with creosote oil or pentachlorophenol. All timber piles should be treated.

(2) Concrete Piles

The two principal types of concrete piles are cast-in-place and precast. Depending on the type of concrete pile selected and the foundation conditions, the load carrying ability of the pile can be developed in either skin friction or point bearing or a combination of both. Generally, concrete piles are employed as a displacement type of piling.

Plain or reinforced concrete piles embedded in the earth are generally not vulnerable to deterioration. The water table does not affect their durability provided it is free of deleterious substances such as acids, alkalis, or chemical salts. Concrete piles extending above the surface of a body of water are subject to damage from the abrasive action of floating objects; from ice where it exists or also from sand scouring. Deterioration can also result from frost action particularly in the splash zone and from internal corrosion of the reinforcement causing spalling of the concrete. Generally, reinforced concrete piles are more susceptible to spalling failure than prestressed piles because of inherent fine cracks in the concrete that develop from shrinkage, handling, placement, and loading. Prestressing, which precompresses the pile element, essentially eliminates open cracks in concrete piles. The durability of all concrete piles is increased by decreasing the aggregate porosity and by increasing the concrete density and concrete cover over the reinforcing steel. WisDOT does not currently use this pile type.

A. Cast-In-Place Concrete Piles

Cast-in-place (CIP) concrete piles can be divided into two main types; those that are formed by driving a steel shell casing and filling it with concrete (cased) and those that are placed uncased. Cased cast-in-place are the most positive type since inspection of the pile shell prior to placing the concrete is possible and more accurate control of concrete placement is attainable.

Uncased concrete piles (also called auger-cast) may be more economical; however, there is a greater inherent risk in their installation from the quality control standpoint and are not generally used. There is no method to

determine the pile capacity during construction of auger-cast piles.

Cast-in-place cased piles can be divided into three categories which are tapered, cylindrical and steel pipe piles. The piles are formed by pouring concrete into closed-end casings which have been previously driven into the ground. A flat, oversize plate is typically welded to the bottom of cased, CIP piles. The formed encasement is a light gage metal shell driven with a mandrel or a steel pipe driven directly. A thin shelled encasement must be carefully evaluated so that it does not collapse from soil pressure or deform from adjacent pile driving. Deformities or distortions in the pile shell could constrict the flow of concrete into the pile producing voids which would greatly reduce pile capacity. It is standard construction practice to inspect the encasement prior to placing the concrete. Care must be exercised to avoid intermittent voids along the pile during concrete placement.

Cast-in-place piles are considered displacement type piles because the majority of their loading is typically transferred by skin friction. Cast-in-place piles are selected over timber piles due to their ability to be driven beyond the minimum penetration depth for a high capacity. Cast-in-place piles are frequently employed in slow flowing streams and areas requiring lengths of 50-120 feet (15 to 37 meters). Current WisDOT design practice for CIP piles computes allowable loads by the following equation:

$$P = 0.40 \times A_p \times F_y' \text{ where}$$

A_p = End area of the pile

F_y' = Concrete compressive strength

Pile capacities are maximums, based on an assumed compressive concrete strength of 3000 psi with a reduction factor of 0.40-0.45 (with slight rounding). The somewhat low concrete strength value of 3000 psi is based on the construction difficulties and unknowns of placement. The Geotechnical Site Investigation Report must be used as a guide in determining actual allowable capacities. Any strength contribution associated with the steel shell is neglected in CIP pile design.

Based on this equation, current practice is to design for 55 tons (490 kN) per pile on 10.75 inch (273 mm), 65 tons (580 kN) per pile on 12.75 inch (320 mm), and 80 tons (710 kN) on 14 inch (356 mm) diameter cast-in-place concrete piles. The minimum shell thickness is 0.219 inches (5.5 mm) for straight steel tube and 0.1793 inches (4.5 mm) for fluted steel shells, unless otherwise noted in the Geotechnical Site Investigation Report and stated in the project plans. Trestle piling shall not be less than 12 inches (300 mm) in diameter.

Steel shells for cast-in-place concrete piles can be readily spliced to accommodate variations in required pile lengths by welding as shown on Standard 11.1. All shells for concrete piling up to 50 feet (15 meters) in length are to be furnished with not more than three shop or field welded splices and up to four welded splices for longer piles unless otherwise stated in the project. The minimum allowable wall thickness is .219 inches (5.5 mm). Where cobbles are present, the minimum wall thickness should be increased to .25 inches (6.5 mm) or thicker to facilitate driving without damaging the pile.

B. Precast Concrete Piles

Precast concrete piles can be divided into two main types: reinforced concrete piles and prestressed concrete piles. The piles have parallel or tapered sides and are usually of rectangular or round cross-section. Since the piles are usually cast in a horizontal position, the round cross-section is not common because of the difficulty involved in filling a horizontal cylindrical form. Because of the somewhat variable subsurface conditions in Wisconsin and the need for variable length piles, these piles are currently not used in Wisconsin.

(3) Steel Piles

Steel piles are generally used for end-bearing piles and typically employ what is known as the HP-section. Steel H-piles are rolled sections with wide flanges such that the depth of the section and the width of the flanges are about equal dimensions. The cross-sectional area and volume displacement are relatively small and as a result, H-piles can be driven through compact granular materials and slightly into soft rock. Also, steel piles have little or no effect in causing ground swelling or raising of adjacent piles. Because of the small volume of H-piles, they are considered 'non-displacement' piling.

Since granular soils are largely incompressible, the principal action at the tip of the pile is lateral displacement of soil particles. Although it is an accepted fact that steel piles develop extremely high loads per pile when driven to end-bearing on rock, some misconceptions still remain that H-piles cannot function as friction piles. Load tests indicate that steel H-piles can function quite satisfactorily as friction piles in sand, sand-clay, silt-and-sand, or hard clay, but will typically drive to greater depths than displacement piles. The surface area for pile frictional computations is considered to be the projected 'box area' of the H-pile, not the actual steel surface area.

In compact sand, there is not significant reduction in intergranular space and no increase in free water; thus, skin friction is not decreased by water lubrication during driving. The pressure of the sand grains against the pile is approximately the

same during driving as it is after driving is stopped. The resulting skin friction is an important source of load carrying capacity.

Clay is compressible to a far greater degree than sand or gravel. As the solid particles are pressed into closer contact with each other and water is squeezed out of the voids, only small frictional resistance to driving is generated by the lubricating action of the free water. However, after driving is completed, the lateral pressure against the pile increases due to dissipation of the pore water pressures. This causes the fine clay particles to increase adherence to the comparatively rough surface of the pile. Load is transferred from the pile to the soil by the resulting strong adhesive bond. In many types of clay this bond is stronger than the shearing resistance of the soil.

In hard, stiff clays containing a low percentage of voids and pore water, the compressibility is small. As a result, the amount of displacement and compression required to develop the pile's full capacity are correspondingly small. As an H-pile is driven into stiff clay, the soil trapped between the flanges and web usually becomes very hard due to the compression and is carried down with it. This trapped soils acts as a plug and the pile acts more as a displacement pile.

In cases where loose soils are encountered, considerably longer end-bearing steel piles are required to carry the same load as relatively short displacement-type piles. This is because a displacement-type pile, with its larger cross-section, produces more compaction as it is driven through materials such as soft clays or loose organic silt.

H-piles are available in many sizes and lengths. Unspliced pile lengths up to 140 feet (42 meters) and spliced pile lengths up to 230 feet (70 meters) have been driven. The most optimum pile lengths range from 40-120 feet (12 to 37 meters). Common H-pile sizes vary between 10-14 inches (0.25-0.36 meters). Allowable load capacities of H-piles are (normally) based on AASHTO design stresses of 6,000 psi (40 MPa) where boulders are present; 9,000 psi (60 MPa) for compact soils with soft rock; up to a maximum of 12 Ksi (80 MPa) if bearing is on sound rock; and up to a maximum of 16 Ksi (110 MPa) if load tests are used to confirm pile bearing on sound rock. WisDOT typically designs steel piles for 9,000 psi design steel stress. The allowable steel design stress should be based on the subsurface conditions and presented in the Geotechnical Site Investigation Report.

Steel H-piles are relatively easy to splice by welding and the detail is shown on Wisconsin bridge plans (Standard 11.1). Current construction practice is to weld diamond plates to the driven pile first; then the diamond plates act as a guide for the upper section of pile. The pile splice is welded from the inside and the diamond plates are fillet welded to the upper pile. Splices are designed to develop the full strength of the pile section. Steel piling 20 feet (6 meters) or less in length is to be furnished in one unwelded piece. Piling from 20-50 feet (6 to 15 meters) in length

can have two shop or field splices and piling over 50 feet (15 meters) in length can be furnished with up to a maximum of four splices, unless otherwise stated in the project plan documents.

(4) Oil Field Pipe

The oil industry uses a very high quality pipe in their drilling operations. Every piece is tested for conformance to their standards. Oil field pipe is accepted as an end-bearing alternate to HP piling, provided the material in the pipe meets the requirements of ASTM A252, Grade 3, with a minimum tensile strength of 66 Ksi (455 MPa). The weight and area of the pipe shall be approximately the same as the HP piling it replaces. Sufficient bending strength shall be provided if the oil field pipe is replacing HP piling in a pile bent. Oil field pipe is driven open-ended and not filled with concrete. The availability of this pile type varies and is subject to changes in the oil industry.

(5) Pile Points and Preboring

A study was made on the value of pile tips (pile points) on steel piles when driving into rock. The results indicated that there was very little difference between the piles driven with pile tips and those without. The primary advantages for specifying pile points are for penetrating or displacing boulders, or driving through dense granular materials and hardpan layers. Piling can generally be driven faster and in straighter alignment when points are used.

Conical pile points have also been used for round, steel piling (friction and end-bearing) in certain situations. These points can also be flush-welded if deemed necessary.

Pile points and preboring are sometimes confused. They are not interchangeable. Points can be used to help drive piles through soils that have gravel and/or cobbles or present other difficult driving conditions. They can also be used to get a good 'bite' when ending piles on sloping bedrock surfaces. Points cannot be used to ensure that piles penetrate into competent bedrock. They may assist in driving through weathered zones of rock or soft rock, but will generally not be effective when penetration into hard rock is desired. If embedment into rock is required or minimum pile penetration is doubtful, preboring should be considered.

(6) Drilled Shafts

Drilled shafts (caisson piles) vary in diameter from approximately 2.5 to 10 feet (0.75 to 3.0 meters). Caissons with diameters greater than 6 feet (2 meters) are generally referred to as piers. Drilled shafts are used to carry large foundation loads to bearing on dense layers or bedrock. They are also cost effective in water crossings with very shallow bedrock, where cofferdams are difficult or

expensive to construct. Shafts may be designed as friction, end-bearing or a combination of both. The drilled shaft may be cased or uncased depending on the soil and depth of bearing.

Drilled shafts have been used on only a small number of structures in Wisconsin. For unusual site conditions, the use of drilled shafts may be advantageous. Design methodologies for drilled shafts can be found in FHWA design manuals.

11.3 PILE EXAMPLE PROBLEM

Given: Geotechnical Site Investigation Report. Determine the pile lengths for the two given pile types. Required pile capacity is 55 tons/pile.

Pile types: 10 ¾-inch diameter Cast-in-Place pile
10 x 42 H-pile

Subsurface Conditions: The base of the proposed footing is located at the existing ground elevation. The following describes the subsurface conditions in the boring:

10 feet of firm clay, overlying
30 feet of loose sand, overlying
10 feet of dense sand, overlying
Limestone bedrock

Groundwater is located at the interface of the clay and sand, 10 feet below the ground surface. Table 1 shows the soil parameters used to calculate pile lengths and capacities.

Typical Table of Subsurface Conditions from Site Investigation Report

Table 1: Soil Parameters and Foundation Capacities						
Soil Description	Friction Angle (degrees)	Cohesion (psf)	Unit Weight (pcf)	Skin Friction ¹ (psf)	End Bearing ¹ (psf)	
					Displacement	Non-Displacement ⁴
Clay 0' – 10'	0	1,000	114	400	4,500	3,000
Sand, loose ² 10' – 40'	30	0	120	230	6,500	4,300
Sand, dense 40' – 75'	35	0	125	720	53,800	35,900
Limestone Bedrock 75' and below	0	96,000	140	NA ³	Refusal ⁵	Refusal ⁵
1. Skin friction and end bearings values have a factor of safety of 2. 2. Groundwater located at the top of this sand layer. 3. NA - not applicable. 4. Non-displacement End Bearing values have been decreased by an additional 33 percent. 5. Refusal, the capacity of the bedrock exceeds the maximum design stress for a steel pile.						

The Table 1 design values are based on the following equations from the 1993, FHWA Soils and Foundations Workshop Manual (SFWM).

Skin Friction*Cohesive Soils:*

$$Q_s = \Sigma(C_a \times C_d \times D) \text{ for each soil layer}$$

Granular Soils:

$$Q_s = \Sigma(K_\delta \times C_F \times P_d \times \sin \delta \times C_d \times d) \text{ for each soil layer}$$

Point Bearing*Cohesive Soils:*

$$Q_p = 9 \times C_u \times A_p$$

Granular Soils:

$$Q_p = A_p \times \alpha \times P_d \times N'_q \text{ Where } Q_p \leq Q_{lim}$$

Determine Pile Lengths (Using design values presented in Table 1.)10 3/4-inch CIP Pile Length for Allowable Bearing Capacity of 55 tons

$$\begin{aligned} C_d &= \text{Diameter} \times \pi \\ &= 10.75'' \times 1 \text{ foot}/12'' \times \pi = 2.814 \text{ feet} \end{aligned}$$

$$\begin{aligned} A_p &= (\text{Radius})^2 \times \pi \\ &= (10.75''/2)^2 \times 1 \text{ foot}^2/144 \text{ inch}^2 \times \pi = 0.63 \text{ feet}^2 \end{aligned}$$

$$\text{Now, } Q_a = \Sigma Q_s + Q_p$$

Total capacity at a depth of 10 feet -

$$Q_a = [(10 \text{ feet} \times 2.814 \text{ ft} \times 400 \text{ psf}) + (0.63 \text{ feet}^2 \times 4,500 \text{ psf})] / 2,000 \text{ pounds/ton} = 7.0 \text{ tons}$$

Total capacity at a depth of 40 feet -

$$Q_a = [(10 \text{ feet} \times 2.814 \text{ ft} \times 400 \text{ psf}) + (30 \text{ feet} \times 2.814 \text{ ft} \times 230 \text{ psf}) + (0.63 \text{ feet}^2 \times 6,500 \text{ psf})] / 2,000 \text{ pounds/ton} = 17.4 \text{ tons}$$

Total capacity at a depth of 74.9 feet -

$$\begin{aligned} Q_a &= [(10 \text{ feet} \times 2.814 \text{ ft} \times 400 \text{ psf}) + (30 \text{ feet} \times 2.814 \text{ ft} \times 230 \text{ psf}) + (34.9 \text{ feet} \times 2.814 \\ &\text{ft} \times 720 \text{ psf}) + (0.63 \text{ feet}^2 \times 53,800 \text{ psf})] / 2,000 \text{ pounds/ton} = 67.6 \text{ tons} > 55 \text{ tons} \\ &\text{(Therefore pile tip is between depths of 40 and 74.9 feet.)} \end{aligned}$$

Calculate **L**

$$55 \text{ tons} = [(10 \text{ feet} \times 2.814 \text{ ft} \times 400 \text{ psf}) + (30 \text{ feet} \times 2.814 \text{ ft} \times 230 \text{ psf}) + (L \text{ feet} \times 2.814 \text{ ft} \times 720 \text{ psf}) + (0.63 \text{ feet}^2 \times 53,800 \text{ psf})] / 2,000 \text{ pounds/ton}$$

$$L = 22.4 \text{ feet}$$

$$\text{Total Length of CIP pile required} = 10 \text{ feet} + 30 \text{ feet} + 22.4 \text{ feet} = 62.4 \text{ feet}$$

10 x 42 H-Pile Length for Allowable Bearing Capacity of 55 tons

$$\begin{aligned} C_d &= 2(\text{flange width}) + 2(\text{section height}) \\ &= [2(10.07'') + 2(9.70'')] \times 1\text{foot}/12'' = 3.294 \text{ feet} \end{aligned}$$

$$A_p = A_{\text{steel}} = 12.4 \text{ inch}^2 \times 1 \text{ foot}^2 / 144 \text{ inch}^2 = 0.086 \text{ feet}^2$$

$$\text{Now, } Q_a = \Sigma Q_s + Q_p$$

Total capacity at a depth of 10 feet -

$$Q_a = [(10 \text{ feet} \times 3.294 \text{ ft} \times 400 \text{ psf}) + (0.086 \text{ ft}^2 \times 3,000 \text{ psf})] / 2,000 \text{ pounds/ton} = 6.7 \text{ tons}$$

Total capacity at a depth of 40 feet -

$$Q_a = [(10 \text{ feet} \times 3.294 \text{ ft} \times 400 \text{ psf}) + (30 \text{ feet} \times 3.294 \text{ ft} \times 230 \text{ psf}) + (0.086 \text{ ft}^2 \times 4,300 \text{ psf})] / 2,000 \text{ pounds/ton} = 18.1 \text{ tons}$$

Total capacity at a depth of 74.9 feet -

$$\begin{aligned} Q_a &= [(10 \text{ feet} \times 3.294 \text{ ft} \times 400 \text{ psf}) + (30 \text{ feet} \times 3.294 \text{ ft} \times 230 \text{ psf}) + (34.9 \text{ feet} \times 3.294 \\ &\text{ft} \times 720 \text{ psf}) + (0.086 \text{ ft}^2 \times 35,900 \text{ psf})] / 2,000 \text{ pounds/ton} = 60.9 \text{ tons} > 55 \text{ tons} \\ &\text{(Therefore pile tip is between depths of 40 and 74.9 feet.)} \end{aligned}$$

Calculate L

$$55 \text{ tons} = [(10 \text{ feet} \times 3.294 \text{ ft} \times 400 \text{ psf}) + (30 \text{ feet} \times 3.294 \text{ ft} \times 230 \text{ psf}) + (L \text{ feet} \times 3.294 \text{ ft} \times 720 \text{ psf}) + (0.086 \text{ ft}^2 \times 35,900 \text{ psf})] / 2,000 \text{ pounds/ton}$$

$$L = 29.9 \text{ feet}$$

$$\text{Total Length of H-pile required} = 10 \text{ feet} + 30 \text{ feet} + 29.9 \text{ feet} = 69.9 \text{ feet}$$

Summary: Based upon the calculations, the CIP piles will drive to a shorter depth (62.4') then the H-piles (69.9').

11.4 SHEET PILING

Sheet piling consists of interlocking steel, precast concrete or wood pile sections driven side by side to form a continuous unit. Individual pile sections usually vary from 12 to 21 inches (300 to 530 mm) in width allowing for flexibility and ease of installation. The most common use of sheet piling is for temporary construction of cofferdams, retaining walls or trench shoring. The structural function of sheet piles is to resist lateral pressures due to earth and/or water. The steel companies have excellent design references. Reference may also be made to "Sheet Pile Design", in the Bridge Computer Manual. Sheet piling walls generally derive their stability from sufficient pile penetration (cantilever walls). When sheetpile walls reach heights in excess of approximately 15 feet (4.5m), the lateral forces are such that the walls need to be anchored with some form of tieback. Cofferdams depend on pile penetration, ring action and the tensile strength of the interlocking piles for stability. If a sheet pile cofferdam is to be dewatered, the sheets must extend to a sufficient depth into firm material to prevent a "blow", which is water coming in from below. Cross and other bracing must be adequate and placed as quickly as excavation permits. Sheet piling is generally chosen for its efficiency, versatility, and economy. Cofferdam sheetpiling and any internal bracing is designed by the Contractor, with the design being accepted by the Department. Other forms of temporary sheetpiling are designed by the Department. Temporary sheetpiling is not the same as temporary shoring. Temporary shoring is designed by the Contractor and may involve sheetpiling or other forms of excavation support.

(1) Common Types of Sheet Piles

Although sheet piling can be composed of timber or precast concrete members, these material types are seldom, if ever, used on Wisconsin transportation projects.

Steel sheet piles are by far the most extensively used type of sheeting in temporary construction because of their availability, versatility and high salvage value. Also, they are very adaptable to permanent structures such as bulkheads, seawalls and wharves if properly protected from salt water.

Sheet pile shapes are generally Z, arched or straight webbed. The Z and the medium to high arched sections have high section moduli and can be used for substantial cantilever lengths or relatively high lateral pressures. The shallow arched and straight web sections have high interlocking strength and are employed for cellular cofferdams. The Z-section has a ball-and-socket interlock and the arched and straight webbed sections have a thumb-and-finger interlock capable of swinging 10 degrees. The thumb-and-finger interlock provides high tensile strength and considerable contact surface to prevent water passage. Continuous steel sheet piling is not completely waterproof, but does stop most water from passing through the joints. Steel sheet piling sections are usually 3/8 to 1/2 inch (9 or 13 mm thick). Designers should specify the required section modulus and embedment depths on the plans, based on corrosion resistance and bending requirements.

Refer to steel catalogs for typical sheet pile sections. Contractors are allowed to choose either hot or cold rolled steel sections meeting the specifications. Previously used steel sheet piling may be adequate for some temporary situations, but should not be allowed on permanent applications.

(2) Driving of Sheet Piling

All sheets in a section are set, if practical, before any are driven. There is a tendency for sheet piles to lean in the direction of driving producing a net "gain" over their nominal width. Most of this "gain" can be eliminated if the piles are driven a short distance at a time, say from 6 feet (2 meters) to one third of their length before any single pile is driven to its full length. During driving if some sheet piles strike an obstruction, move to the next pile that can be driven; then return to the piles that resisted driving. With interlock guides on both sides and a heavier hammer, it may be possible to drive the obstructed sheet to the desired depth.

Sheet piles are installed by driving with gravity, steam, air or diesel powered hammers, or by vibration, jacking or jetting depending on the subsurface conditions, and pile type. A vibratory or double acting hammer of moderate size is best for driving sheet piles. For final driving of long heavy piles a single acting hammer may be more effective. A rapid succession of blows is generally more effective when driving in sand and gravel; slower, heavier blows are better for penetrating clay materials. For efficiency and impact distribution, where possible, two sheets are driven together. If sheets adjacent to those being driven tend to move down below the required depth, they are stopped by welding or bolting to the guide wales. When sheet piles are pulled down deeper than necessary by the driving of adjacent piles, it is generally better to fill in with a short length at the top, rather than trying to pull the sheet back up to plan location.

(3) Pulling of Sheet Piling

Sheet piles are pulled with air or steam powered extractors or inverted double acting hammers rigged for this application. Also, vibratory hammers are effective in pulling. Occasionally a derrick with multiple-reefed blocks is needed. If piles are difficult to pull, slight driving is effective in breaking them loose. Pulled sheet piling is to be handled carefully since they may be used again; perhaps several times.

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